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Evaluating Hydraulic Performance of Two District Water Distribution Networks in Baghdad City

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ABSTRACT

This study evaluates the hydraulic performance of Baghdad's water supply network by analysing pressure heads and velocities. In this work, Water-CAD hydraulic models integrated with QGIS were utilized. The districts 821 and 835 were selected in partnership with Al-Rashid Municipality Water Department, two districts fed from a single source, with the construction period of the two networks differing. They were calibrated in Darwin calibration using field-measured pressure at five and three locations, respectively, achieving coefficient of determination (R^2) = 0.99. In District 821, pressure analysis during peak demand revealed that most junctions operated under 7 m H_2O, with 14% exhibiting negative pressure and 84% below this threshold. Velocity analysis showed that 94% of pipes-maintained velocities below 0.5 m/s, 5.6% operated between 0.5–2 m/s, and 0.5% exceeded 2 m/s. At low demand, more than half of the junctions recorded pressure heads below 7 m H_2O, while 25% exceeded this value. Velocity levels during low demand do not have considerable variations compared with peak hours usage. In district 835, all junctions recorded pressures below 7 m H_2O during peak demand, with about 50% experiencing negative pressure. Low demand analysis indicated 80 junctions operated below 7 m H_2O, while 27 exceeded this criterion. Velocity analysis revealed that 83% of pipes operated below 0.5 m/s, with the remaining 17% within 0.5–2 m/s. In conclusion, both networks predominantly operated below the acceptable pressure head of 7 m H_2O, highlighting insufficient water supply to consumers, particularly during peak demand periods.

Keywords: Darwin calibration, QGIS, Hydraulic analysis, Water distribution networks, Water-CAD.

1. INTRODUCTION

Water distribution networks (WDNs) are an essential part of any city or region's infrastructure. These networks aim to deliver clean, potable water to homes, institutions, and public facilities in adequate quality and quantity **(Alsaydalani, 2019; Kuma and Abate,**

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2021; Munawar, 2023). Water distribution networks consist of a combination of pipes, valves, tanks, and pumps that work together to ensure water supply to all users sustainably and efficiently **(Alannz, 2018; Al-Fatlawi and Merjan, 2019)**. Many factors in the water distribution network can affect the network performance in several ways, notably the age of infrastructure, water demands due to increasing population or activity, which may affect the water pressure and quality of water used. For these reasons, it needs periodic evaluation to ensure good performance and the quality of the water distributed **(Albadry, 2017)**. Hydraulic WDNs assessment involves determining the flow velocity and its even distribution, measuring water pressure at different points in the network, and monitoring pressure loss across the network. Many programs are used to analyse and evaluate water distribution networks. One of the common programs used to model and analyse water distribution networks is Water-CAD **(Sonaje and Joshi, 2015)**.

Many previous studies have been conducted on evaluating water distribution networks on the International Level**, (Dave et al., 2015)** studied the analysis of the water distribution system WDS in Gandhinagar City using EPANET Software. The analysis was carried out after estimating water demand for three decades by forecasting the population of the city in three methods, namely geometrical, arithmetic, and incremental increase methods. The EPANET Software data analysis results reveal minimal head loss, which is crucial for maintaining the continuous pressure needed to ensure an uninterrupted water supply system. **(Agunwamba et al., 2018)** used Water-CAD and EPANET software to assess the performance of the water distribution system in the Wadata sub-zone. The assessment involved analyzing pressure, velocity, hydraulic head loss, and nodal demands. It was found that EPANET produced slightly higher pressure and velocity results in about 60% of the cases examined. However, around 19% of the total number of nodes analyzed had negative pressures, indicating that there is insufficient head within the distribution network for water conveyance to all the sections. Additionally, 69% of the nodes had pressures less than the adopted pressure for the analysis. This means that the water supply may not be sufficient for certain areas. When it comes to the velocity of the flow in the pipes, around 88% of the flow velocities were within the adopted velocity, while about 12% of the velocities exceeded the adopted velocity. The results of this study detected inefficient performance of Wadata sub-zone water distribution system under demand.

The **(Belachew et al., 2021)** studied evaluating the hydraulic performance of Asella town's water supply distribution system with respect to velocity and pressure using Water GEMS v8i software. They considered that the average daily consumption per capita at the time of the application study was 35.3 l/d per person. The results of the extended period simulation at peak and minimum hourly consumptions were found to be 47.08% and 10%, respectively, for a pressure value of 60 m. Also, the flow velocity of the pipes at maximum hour consumption recorded 79.56% of the velocity 2 m/s. from the 650 nodes, showing that 350 nodes were at a critical point due to pressure head less than 15 m (at high elevations and far from the supply points) and to deliver water with a required pressure. They recommended two solutions to modify the amount of water, the first was adding a new water source and the second was increasing the pipe diameter or adding pipes in parallel. **(Kassahun and Dargie, 2024)** aimed to evaluate the performance of the existing water supply distribution system using Water GEMS software. In in this work, the hydraulic modelling of the system was treated as a continuous supply system. Calibration was carried out by using the observed pressure data to ensure the model performance, and an extended period simulation EPS was employed for the analysis. The current system outputs indicated that pressure is very high (above the maximum value) and velocity is very low during peak hour demand. Before optimization, the performance of the system had been negatively impacted by the velocity and pressure results.

On the National Level, **(Kadhim et al., 2021)** attempted to develop a geographic information system model to manage water distribution networks in Al-Karada region by assessing the network geographically in a GIS application. The results of the network analysis showed that no errors were found in the design and management of the network by using tools that identified the network system problems (such as pipe breaks) immediately and followed rapid maintenance solutions to optimize the water network. **(Hussein et al., 2021)** highlighted studying the efficiency of one of the main water pipes in the Karbala region by evaluating and improving it. For this purpose, a fieldwork reading at different junction locations was conducted to analyze variations in water losses. This study analyzed the main pipe by dividing it into two sections for assessment. The first was analyzing the collected data with the study state flow assumption, and the second part dealt with hourly consumption variation by extended period simulation analysis in the water-CAD software. The results of the first part showed large losses of water at the beginning of the main water pipe, starting from junction-1 to junction-5, while large scarcity was observed at the end of the main water pipe. They found that the entire losses and deficiency in the main pipe were equal to 326.24 and 113.25 m^3/hr., respectively, which means the quantity of losses is much more compared to the quantity of scarcity. They concluded that if the quantities of losses were controlled, the quantities supplied to the main water pipe would be enough to fill the presence of scarcities without the need to increase the quantities supplied based on the steady-state flow assumption, without any future expansions for junctions. (**Kadhim et al., 2021)** evaluated the water distribution network of Al-Karada district, Baghdad, Iraq. The study calculated and reviewed values of pressure and velocity under the assumption of a daily requirement per capita of 350 liter/day. The study established that the flow velocity is not high and pressures are acceptable at 0.7-2.2 bar. **(Abdulsamad and Abdulrazzaq, 2022)** studied calibration and analysis Al-Yarmouk water network using ArcMap 10.8 and water-GEMS. They created a hydraulic network model to simulate the network. The pressures at eight different junctions were obtained using the Bourdon gauge to calibrate the model. The steady-state time at average demand served as the basis for the analysis. With a 0.988 correlation coefficient, the results of the analysis indicated that the network's pressure varied between 8-21 m H_2O. The velocity of the main pipe was between 0.5-2 m/s, which was considered within acceptable limits. It was noted that a decrease in pressure in the pipes that were farthest from the sources of water. In the internal distribution network, an increase in velocity was observed in the main pipes, attributed to low water consumption in the lateral pipes.

This research aims to evaluate the hydraulic performance of two selected water supply networks in Baghdad City by monitoring and calculating the hydraulic parameters (pressure head and velocity) at different locations of the two WDNs using the water-CAD applications.

2. MATERIALS AND METHOD

2.1 Source of Potable Water and Baghdad Water Supply Zones

Baghdad and its districts extend across an area of 950 km^2. Baghdad contains a large water supply network. These networks had been constructed at different times and could be quite old or relatively new. The system incorporates a range of pipe materials, including ductile iron, asbestos cement, PVC, and other types **(Al-Suhaili and Al-Azzawe, 2011; Abdulhamid, 2017)**. The Tigris River is one of two main sources of surface water in Iraq, and it is the only source of drinkable water for Baghdad **(Abdulrazzaq and Kamil, 2010)**. It flows from north to south and divides the capital into two sides - Karkh on the right (Western side) and Rasafa on the left (Eastern side). For the water supply management, Baghdad Water Authority (BWA) divided service areas into 25 water supply zones WSZs, 11 on the Karkh side denoted "K", and 14 WSZs on Rasafa side with "R" (**Fig. 1**).

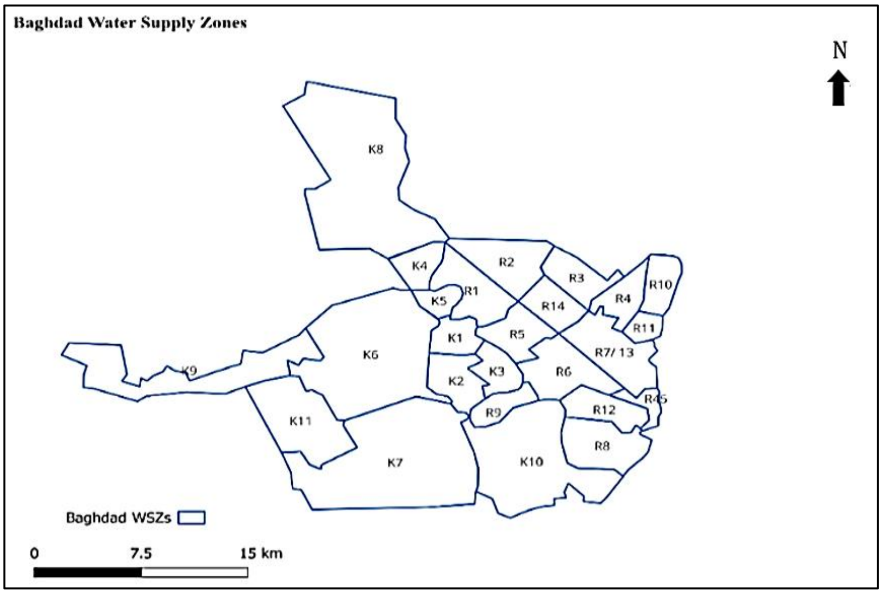


Figure 1. Water supply zones in Baghdad **(JICA, 2006)**.

Each WSZ is comprised of administrative districts called Mahalah, with an estimated total number of 477 Mahalahs in the city. Out of these, 272 Mahalahs are located on the Rasafa side, while 205 are on the Karkh side **(Aenab and Singh, 2012; Khudair, 2013; Raheem and Al-Juhaishi, 2024)**. In this study, two districts, 821 and 835, located within K7 WSZ, were selected for an evaluation of their water networks, as illustrated in **Fig. 2**.

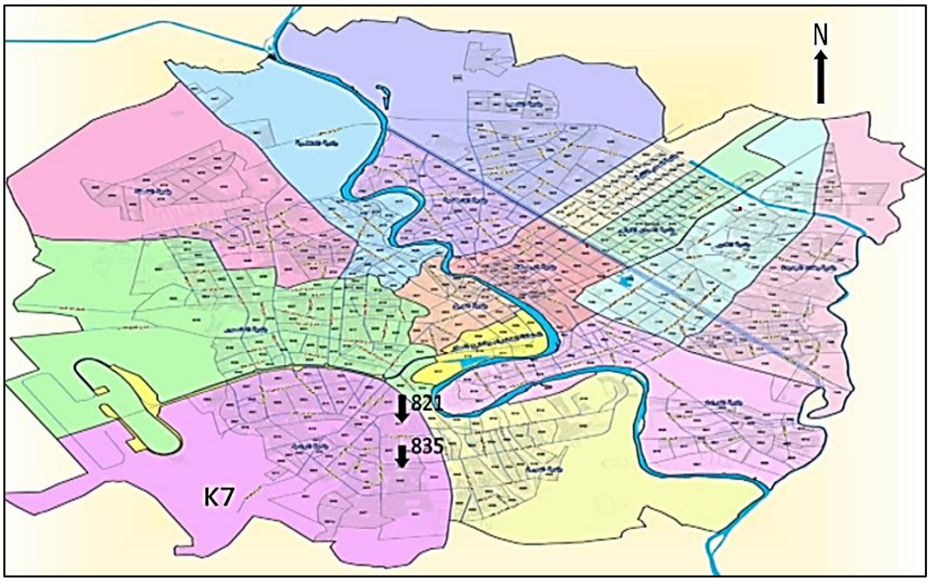


Figure 2. Administrative in Baghdad City **(JICA, 2006)**.

2.2 Study Area

2.2.1 District 821

It is located on the Karkh side of the capital, Baghdad, in the southwest. This district is one of the 49 localities within the municipality of Al-Rashid (K7 WSZ) and the biggest. It has an area of about 2.8 km^2 (according to Google Earth projection) and a population of about 14000 people. The water distribution network in this Mahalla was replaced in 2021 as part of the Baghdad Municipality's plan to renew the water distribution networks, so it is considered one of the modern networks. This WDN is fed from the southern reservoir through a main Ductile Iron (DI) pipe with a diameter of 1400 mm, as shown in **Fig. 3**.

2.2.2 District 835

It is the second network selected as a case study of the 49 localities within Al-Rashid municipality, with an area of about 0.5 km^2 (according to Google Earth projection) and a population of about 5500 people. It is located in the south of the 821 districts in Al-Saydiyah region (**Fig. 4**). The network of district 835 consists of a single reservoir and 156 pipes transporting water to 107 demand nodes by main Ductile Iron pipe with 1000 mm.

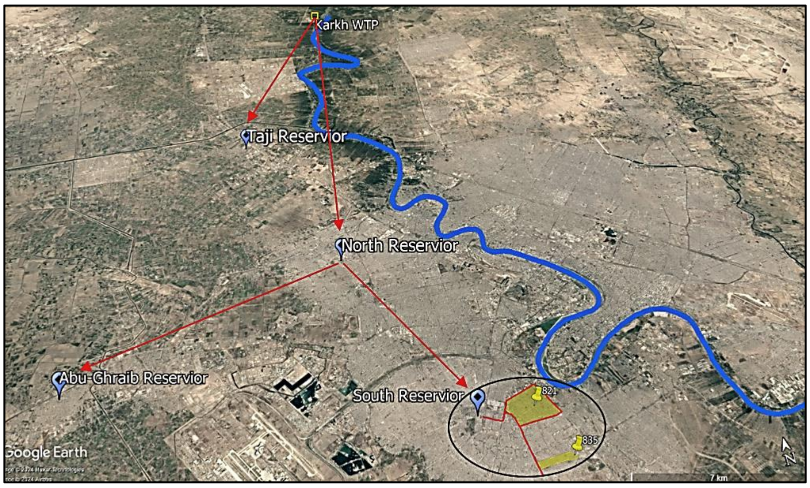


Figure 3. Location of a case study concerning a water source.

Figure 4. Case study zoom-in.

As mentioned above, the methodology was applied to two case studies: the network of district 835 consisted of 156 pipes and 107 nodes, while a network of district 821 comprised 266 nodes and 409 pipes which summarized in **Table 1**, also DI and PVC type with different diameters and lengths are presented in **Table 2**.

Table 1. Summary of the model components of two WDNs.

District No.	No. of junctions	No. of pipes	Reservoir	Pump station
821	266	409	1	1
835	107	156		

Table 2. Summary of pipe diameter and length used in the model of two WDNs.

District no.	821				835			
Pipe type	PVC			Ductile Iron	PVC		Ductile Iron	
Diameters (mm)	200	150	100	>250	150	100	250	>250
Lengths (m)	15,204	12,110	24,285	8,350	1,280	8,500	2,570	3,950

The pipe diameter legend of the two districts is shown in **Figs. 5 and 6**.

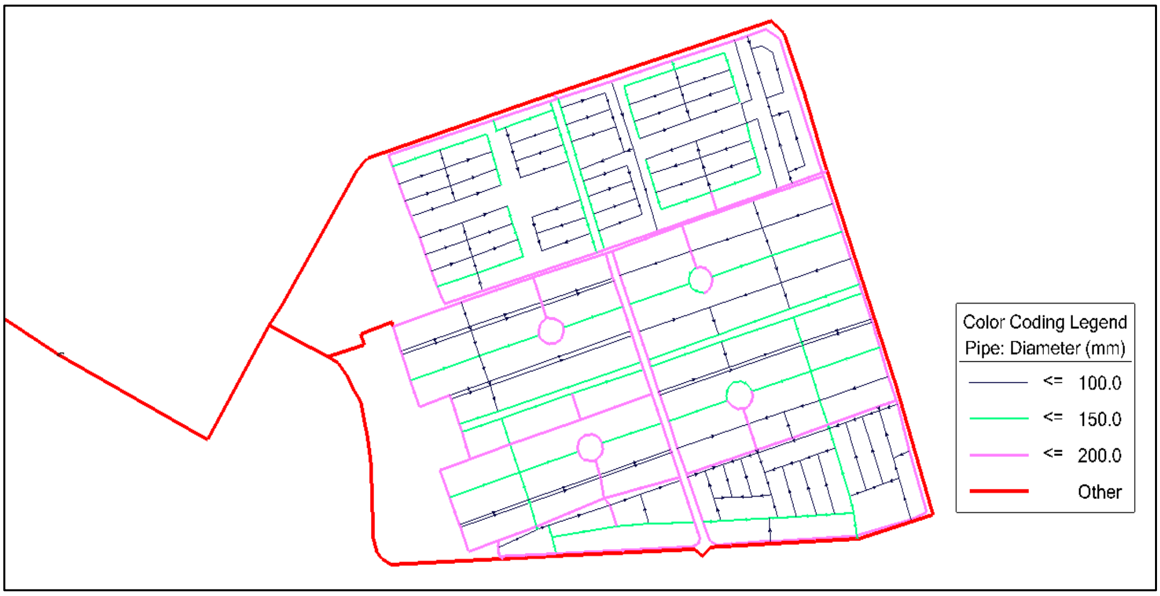


Figure 5. Pipe diameter used in the model of 821 WDN.

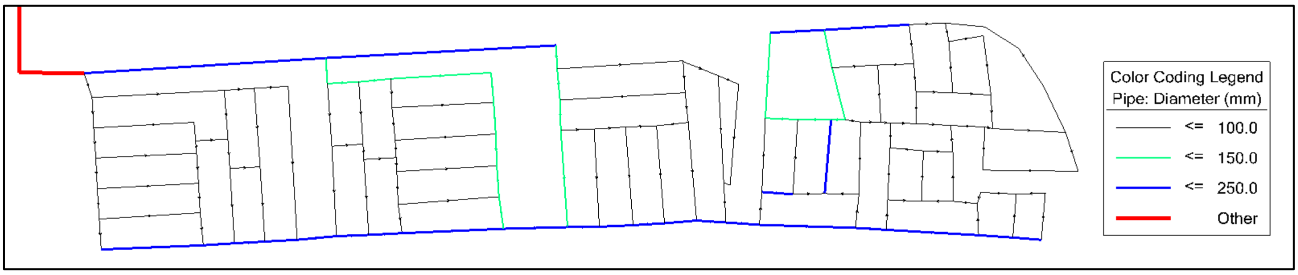


Figure 6. Pipe diameter used in the model of 835 WDN.

2.3 Field Measurements

In this study, fieldwork involved pressure measurements at various points within two networks. The pressure measurements were taken at the selected points of the distribution system as shown in **Fig. 7**. Five and three measurement points were selected in the districts 821 and 835, respectively. Four readings were taken at three different times during two seasons: summer and winter. Measurements were conducted using a glycerin pressure gauge, a commonly used tool for pressure measurement. The gauge was connected to the home's faucet and located near the supply main nodes at most points, as shown in **Fig. 8.**

Figure 7. Pressure field measurements points in two districts.

Figure 8. Field pressure measurement.

2.4 Network Layout

Modelling the water distribution network starts with exporting an existing network. In this study, two original paper maps of WDNs were scanned from BWA and reprojected in QGIS to be laid out in Water-CAD by integrating with a QGIS shapefile. This integration provides visualization for a detailed and intuitive representation of the water distribution network and reduces the need for manual data entry.

2.5 Network Modelling

In this study, the hydraulic performance of the two water distribution systems was analyzed using the Water-CAD hydraulic model integrated with QGIS. Water-CAD can be integrated with external software, like Auto CAD and ArcGIS, and it requires less effort and a shorter time to build a model **(Sonaje and Joshi, 2015; Abdelazim et al., 2023)**.

2.6 Water Demand

The water demand is arguably one of the most important and unpredictable factors within the framework of drinking water supply networks. It refers to the quantity of water needed for different human activities. Water demand varies from one time to another, this depends on many factors such as people's activities, their lifestyle, and climate. It is the first step in modelling. Thus, to model water demand – a first step – must take into account consumption type **(Vertommen et al., 2021)**. Generally, water demands according to consumption type are divided into two parts: domestic and non-domestic demands **(Van Zyl et al., 2017)**.

2.6.1 Water Demand Calculations

Baghdad Water Authority (BWA) classified its consumers' usage into two major categories: domestic use, which occupies 87%, and non-domestic use at 13%. Non-domestic use is classified into two categories: governmental use and commercial & industrial use as shown in **Fig. 9**.

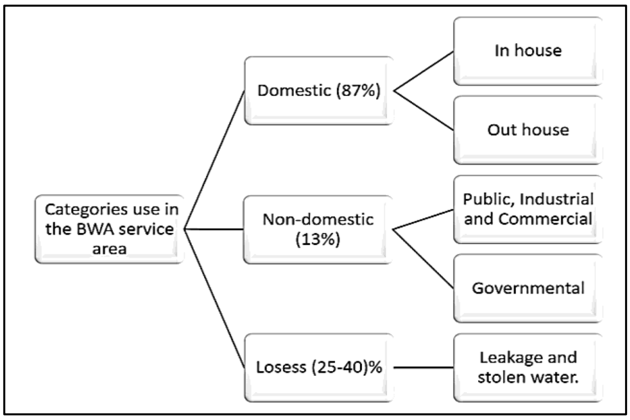


Figure 9. A Summary of the consumer categories in the BWA service area.

With the absence of customer meters and no new population census, the demand consumption cannot accurately be determined, and water consumption is indeterminable, so water demand calculations were provided with approximate needs. In the current study, domestic demand was calculated depending on the estimated number of people in the study area based on the total number of houses data estimated, multiplied by a rate of 5 capita per residential unit and average daily water usage per person of 350 litres per capita per day (LPCD) obtained from Al-Rasheed Municipality water supply office for each district **(JICA, 2006; Al-Anbari et al., 2009; Kamoona et al., 2014; Abboud et al., 2024)**. The steps for calculating Domestic Water Demand (DWD) in two specific areas are outlined below:

- Determine the size of the population in each area.
- Multiplied it by the average daily water usage per person (350 LPCD).

International Plumbing Code (IPC) and International Building Code (IBC) were used to calculate non-domestic water demand (NDD) (schools, mosques, and hospitals **(Code, 2006; Woodson, 2009)**.

In the current study, and addition to the two of the above water uses, the results of two water demand districts are increased by 25% to include the water losses due to any potential water waste **(Abboud et al., 2024)**. In this study, the total average water demand (TAWD)is calculated by the sum of the two above water demands, in addition to losses. It is obtained as follow:

$$(TAWD)= (DWD+ NDD) + WL. \quad (1)$$

Water demand calculations are carried out using the Excel program, as shown in **Table 3.**

Table 3. Summary of demand calculations in two districts.

Total demand calculations (L/d)					
District no.	**Domestic demand**	**Non-Domestic demand**			**Sum.**
		School	**Mosque**	**Hospital**	
821	6,033,125	1,348,974	386,070	238,455	8,006,624
835	2,334,063	563,208	317,940	0	3,215,211

It is not possible to determine the actual water consumption; thus, an approximate calculation has been made with the help of different kinds of water consumption. With absent land use patterns and because detailed demand data is unavailable, the water demand allocation is distributed evenly throughout each junction. This procedure has been adopted in previous studies, which could simplify the initial modelling process **(JICA, 2006)**.

2.7 Model Calibration Process

Before network analysis, calibration should be performed. Calibration of water distribution networks is considered a challenging process and extremely time-consuming, and frustrating to research when dealing with typical water systems due to several reasons, such as the size of the network, the complexity of the network, interference and overlapping service areas, a limited quantity of field data, making it difficult to achieve precise calibration. In this study, the Water-CAD tool calibrated the model, which uses a genetic algorithm to identify the optimal model parameters. According to **(Walski et al., 2003)**, the calibration process involved changing two most sensitive system parameters, the rough coefficient of the pipe type and water demand, until the pressure field data agreed with the model and the criteria elimination of the errors deviations between the results of the model application and the field observations to be within the acceptable **(Ormsbee and Lingireddy, 1997; Rathi et al., 2020) (Table 4)**, **(Figs. 10 and 11)**.

Table 4. Comparison between simulated and measured pressure junctions of two districts.

	Junction no.	Junction label	Measured pressure values (m H_2O)	Simulated pressure values (m H_2O)	RMSE
821	1	J-30	6	5.36	0.33
	2	J-171	5	4.71	
	3	J-199	1	0.98	
	4	J-22	1	1.09	
	5	J-186	1	0.8	
835	1	J-2	3.53	3	0.36
	2	J-43	2.59	2.5	
	3	J-57	0.67	1	

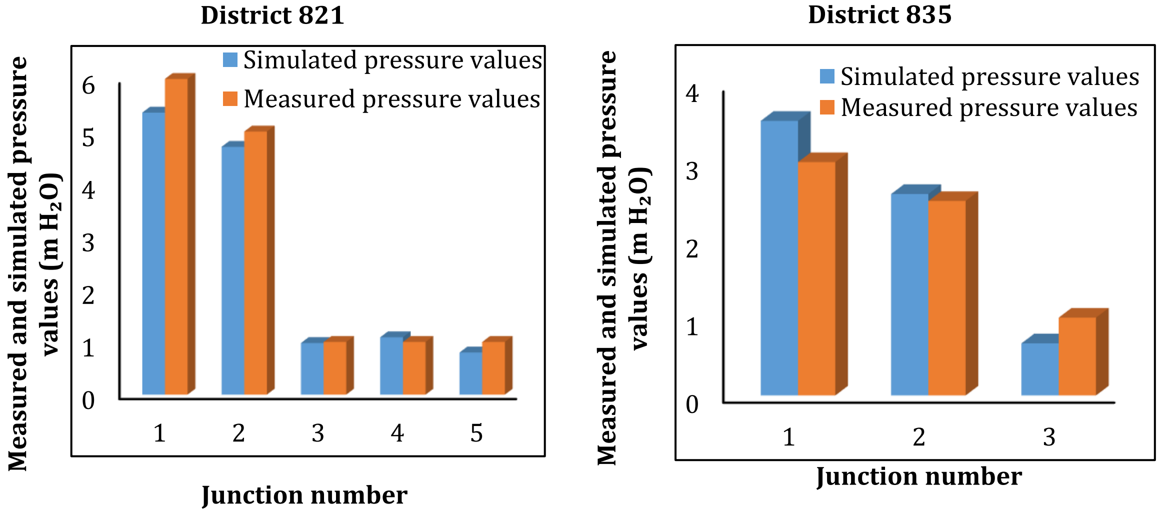


Figure 10. Simulated and measured pressure variation at representative junctions of two districts

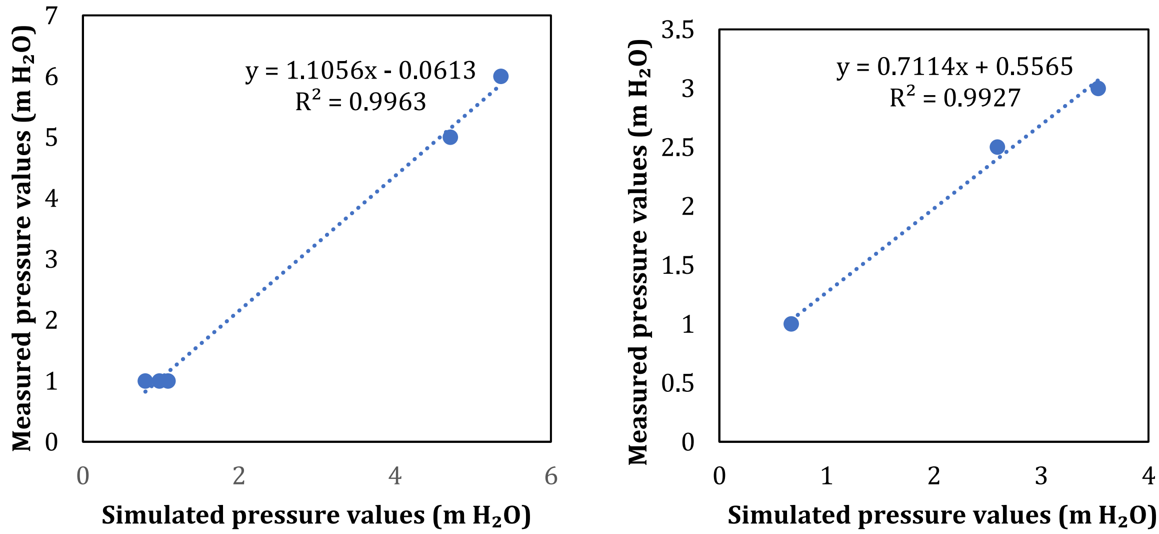


Figure 11. Correlation between simulated and measured pressure of the two districts.

3. RESULTS AND DISCUSSION

3.1 Pressure Head Analysis

The analysis of water supply network pressure and velocity remains crucial, particularly where changes occur in consumption rates **(Basmenji et al., 2017)**. In this study, a pressure heads analysis was performed through hydraulic assessment of distribution networks using Water-CAD software for EPS analyses with a one-hour time step of twenty-four simulation periods at different demand levels. Peak and minimum water demand hours are chosen in the analysis, which were at 12 a.m. and 12 p.m., respectively.

3.1.1 Pressure head analysis in the 821 networks at low demand

The available minimum pressure parameters vary widely around the globe, as shown in **Table 5**.

Table 5. Minimum pressure values around the world (m) **(JICA, 2006; Al-Mousawey, 2022)**.

Country	Region	During all conditions
Iraq	Baghdad	10
USA	Louisiana	10.5
	Connecticut, Oklahoma & Delaware	17.5
	Other states	14
Other countries	UK and Wales	10
	South Africa	24
	Hong Kong	20

In 821 WDN, the inventory dialog box displayed a total node count of 266. In low demand, the analysis of the results showed that the pressure head ranged between 1 and 13.4 m. BWA identified minimum water pressure criteria of 10 m **(JICA, 2006)**. **(Kadhim et al., 2021; Mawlood, 2010)** considered range between 0.71- 2.2 bar under good pressure as compared with the standard value (1.4-1.9) bar for normal use in areas where residential buildings do not exceed 4 floors and are not equipped with fire hydrant units. The Distribution Manual of United Utilities Water Public Limited Company showed that Domestic water-using fittings and appliances function at their best under maximum work pressures not exceeding 10 bar. Domestic appliances typically operate at 0.5 bar minimum pressure; however, certain devices require a minimum of 1.5 bar for proper operation **(Smith, 2015)**. In this study, 7m of water head was used as the minimum pressure measurement. The results showed that 200 nodes out of 266 were less than the minimum acceptable value of pressures. This indicates that 75% of junctions in this network have pressure heads below 7m H_2O during low-demand hours. This area is described in **Fig. 12** below the area highlighted by the aqua colour. While blue colour represented the area that had a pressure head above the minimum adopted pressure of 7 m H_2O, which occupied 25% of the total junctions in the 821 networks. Several reasons for the low water pressure in the distribution network during low-demand hours. Leakage can be suspected in that part of the system.

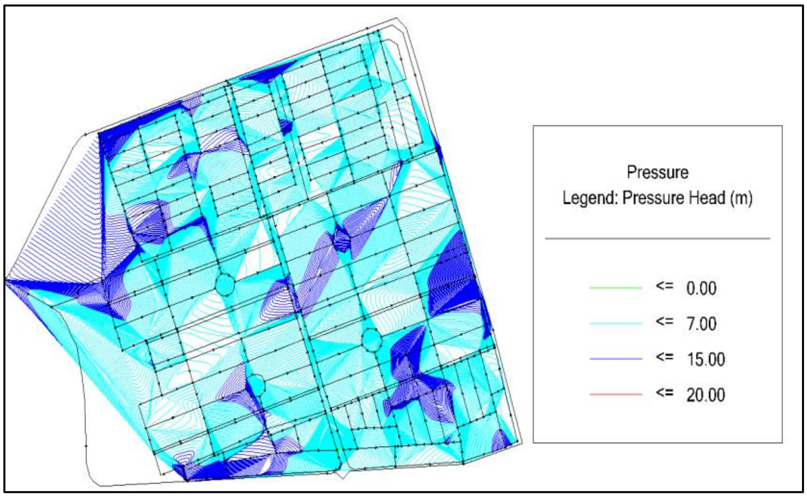


Figure 12. Pressure contour map of junctions at low consumption hours in district 821.

3.1.2 Pressure Head Analysis in 821 Network at Maximum Demand

The results of the pressure head in **Fig. 13** indicate that almost all areas suffer from low pressure during peak demand hours. It found that 98% of junctions that appeared in aqua color were below the minimum pressure head. This percentage included 14% of junctions with negative pressure highlighted by green color. These results are considered a critical indicator that most parts of the network are operating below the acceptable pressure head of 7 m H_2O. This means an insufficient water supply to consumers.

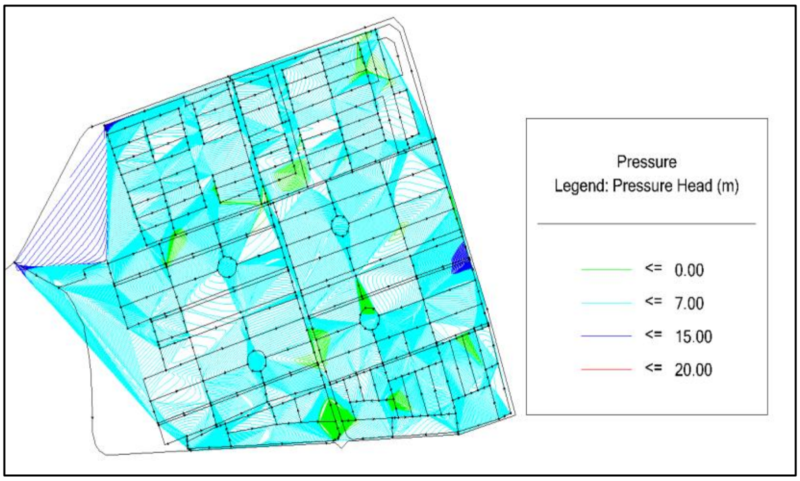


Figure 13. Pressure contour map of junctions at peak consumption hours in district 821.

3.1.3 Pressure head analysis in the 835 networks at low demand

The analysis of the simulation results in the network 835 showed that the pressure heads for 80 nodes out of 107 junctions have a pressure head below the minimum acceptable criterion of 7 m H_2O during low-demand hours, as shown in **Fig. 14**, represented by aqua color in the pressure head legend box. Only 27 junctions maintain a pressure head above 7 m H_2O, which appeared in blue areas.

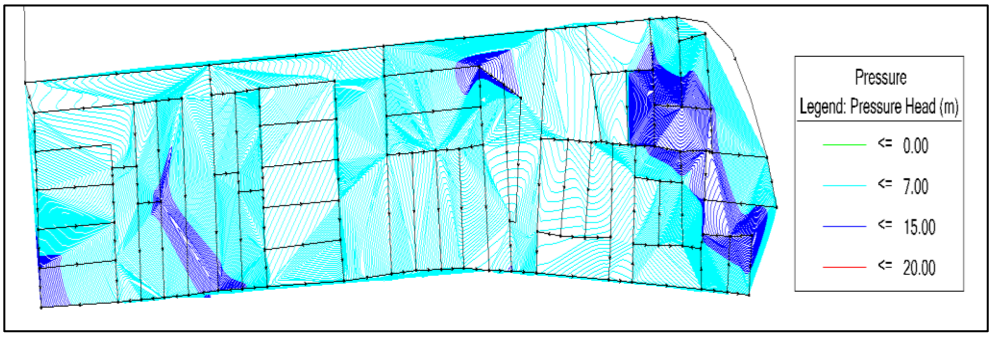


Figure 14. Pressure contour map of junctions at low consumption hours in district 835.

3.1.4 Pressure Head Analysis in 835 Network at Maximum Demand

At maximum consumption, the pressure results analysis indicated all junctions in the network 835 were operated under 7 m H_2O, and about 50% of them with negative pressure. These results are clear in **Fig.15.**

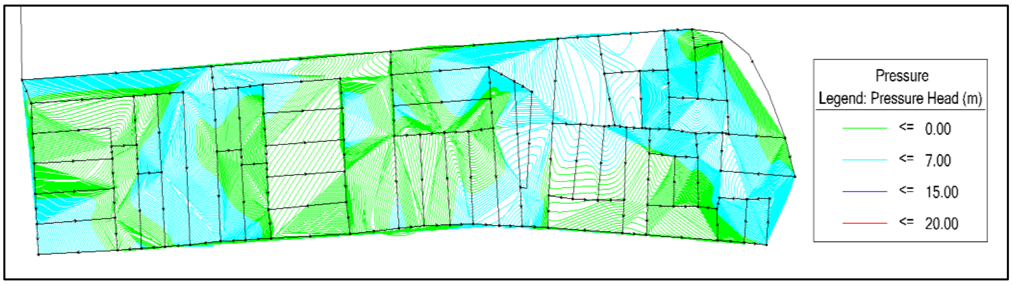


Figure 15. Pressure contour map of junctions at peak consumption hours in district 835.

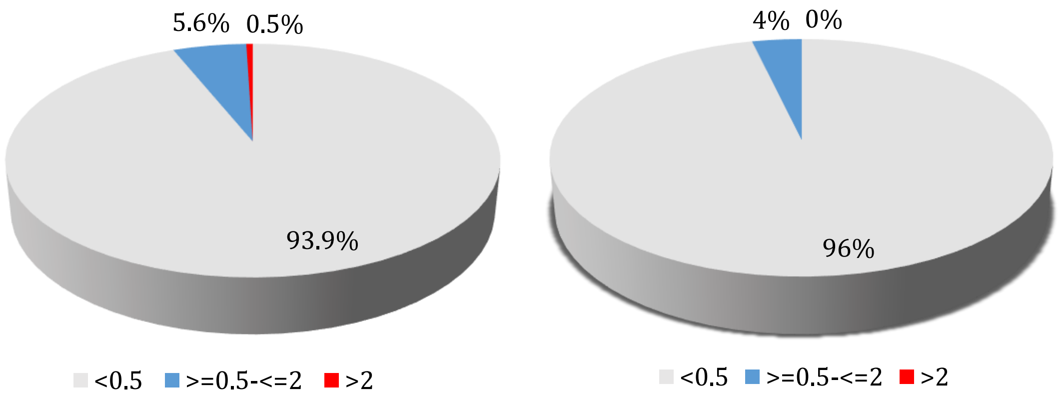
Low pressures reaching customers during periods of low usage may result from network violations, illegal connections to serve newly built residential complexes near the study area that were not part of the original water distribution plan, leakage in some pipelines of the network and insufficient supply from pump station that operate below maximum capacity due to insufficient power supply **(Mawlood, 2010; Al-Mousawey, 2022)**. According to the latest published report from Baghdad Water Department, citizens receive weak water due to the above reasons **(Abboud et al., 2024)**. While in peak period, the negative readings from the nodes show that not all network sections can receive sufficient water flow because the distribution network lacks sufficient head pressure due to resident home pumps operated to overcome the original water supply deficit from the source leads to reduced water pressure heads in addition to the reasons mentioned above **(Agunwamba et al., 2018; Al-Mousawey and Abed, 2023)**. As well as the reasons mentioned above, residential-to-commercial land use pattern changes occur throughout the main streets of district 821.

3.2 Velocity Analysis

The standards of velocity in the water networks differ from one another; this variation can vary based on its specific use within the network. Water velocity should be maintained at less than 2 m/sec in the distribution system and not more than 2.5 m/s in a transmission system **(Belachew et al., 2021)**. Other guidelines recommend that the maximum velocity in water distribution networks should always remain below 3 m/s to prevent pipe corrosion, water hammer effects, and minimize pipe burst **(Al-Mousawey, 2022)**. On the other hand, some standards allow certain pipes to maintain zero velocity, but different standards require (0.4-0.6) m/s as the minimum velocity needed for pipeline sediment prevention. In general, the optimum velocity of flow in pipes of the water network is 1.0 m/s **(Smith, 2015)**. In this study, the velocity was adopted to a 0.5-2 m/s range in analyses.

3.2.1 Velocity Analysis of District 821

The inventory dialog box shows that the total number of pipes in network 821 were 409 pipes, 9 DI with a diameter ≥250 mm, and 400 are PVC pipes with a diameter ≤200 mm. The analysis of the results during peak hours demand shows that about 94% of the pipes have velocities below 0.5 m/s, while 5.6% of pipes are within the range 0.5-2 m/s. For the pipes have velocities more than 2 m/s found 0.5% of the pipes. During low-hour consumption, the velocity analysis in the 821 networks did not differ significantly as compared with peak hour consumption, as shown in **Fig. 16**. It does not appear that pipes with velocities more than 2 m/s and 96% of the pipes network are below 0.5 m/s.

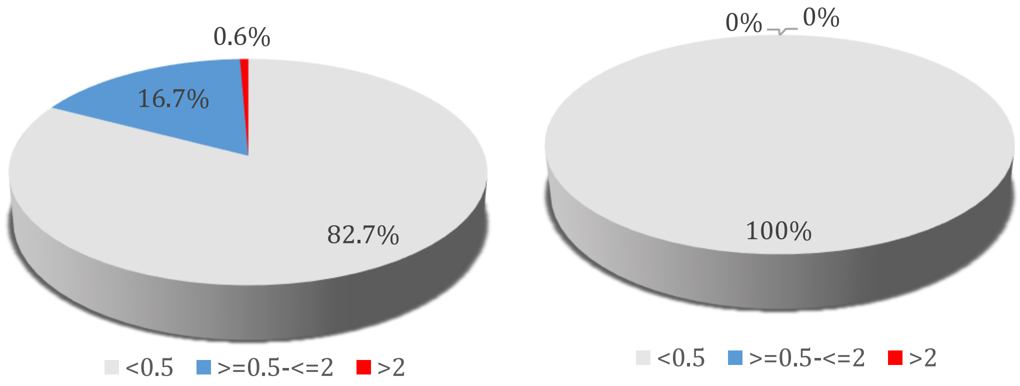


(a) **(b)**

Figure 16. Distribution of velocities at (a) peak and (b) low consumption hours in district 821.

3.2.2 Velocity Analysis of District 835

In the 835 networks, the total number of pipes was 156 pipes, 26 of which were DI with a diameter ≥250 mm, and 130 were PVC pipes with a diameter ≤200 mm. In maximum consumption, all pipe's velocities were below 0.5 m/s, while during low demand, the velocities were divided into three categories: the first group with velocities below 0.5 m/s, which represents about 83% of pipes. The second group consisted of the pipes with the range 0.5- 2 m/s were about 17% of the pipes. These percentages are presented in **Fig. 17** in blue color.



(a) **(b)**

Figure 17. Distribution of velocities at (a)peak and (b) low consumption hours in district 835.

4. CONCLUSIONS

The following conclusions emerged from the analysis of hydraulic simulation results of Water-CAD software with 821 and 835 water distribution networks:

- An analysis of the nodes showed that some measurements reported pressures lower than the required pressure adopted in the study criteria. Not all areas receive enough water from the supply through these measurements.
- Pressures head in the two distribution networks fail at the maximum consumption hour (98% of junctions included 14% of junctions with negative pressure in district 821 and 100% of junctions included 50% of them with negative pressure in district 835) and are lower than the permissible limit at midnight (75% of junctions in two district).
- Most parts of the two networks operate below the acceptable pressure head of 7 m H_2O. This means an insufficient water supply to consumers.
- The two networks face similar issues (low pressure and velocity) despite their different operational durations.
  This means that the principal reason behind this issue stems not only from the networks but also from the source.

According to the results, the authors recommend these points as follows:

- It is necessary to limit illegal connections and other sources of non-revenue water.
- Adopting the SCADA system functions as the main solution to achieve optimal distribution network hydraulic performance.
- Study new scenarios after achieving the required information, such as the fire hydrant effect, pipe resizing, connecting whole networks and suggesting improvements to the two networks in the future.

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Credit Authorship Contribution Statement

Hawraa Mohammed Raheem: Writing the original draft, Software, and Methodology. Mohammed Rashid Al-Juhaishi: Supervision, Validation, Review, Editing, and Proofreading.

Declaration of Competing Interest

The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

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تقييم الأداء الهيدروليكي لشبكتي توزيع مياه في منطقتين بمدينة بغداد

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الخلاصة

تُقيّم هذه الدراسة الأداء الهيدروليكي لشبكة إمداد المياه في بغداد من خلال تحليل شحنة الضغط والسرعات. في هذا العمل، استُخدمت نماذج Water–CAD الهيدروليكية المدمجة مع برنامج QGIS. اختيرت المنطقتان 821 و835 بالشراكة مع دائرة ماء بلدية الرشيد، وهما منطقتان تُغذّيان من مصدر واحد، وتختلف مدة إنشاء الشبكتين لهما. تمت معايرة النماذج الهيدروليكية للمنطقتين في معايرة داروين باستخدام الضغط المقاس ميدانيًا في خمسة وثلاثة موقع على التوالي، محققة معامل تحديد (R^2) يساوي 0.99. في المنطقة 821، كشف تحليل الضغط أثناء ذروة الطلب أن معظم العقد تعمل تحت ضغط ماء 7 متر، حيث أظهر 14٪ ضغطًا سلبيًا و84٪ أقل من هذه العتبة. أظهر تحليل السرعة أن 94٪ من الأنابيب حافظت على سرعات أقل من 0.5 متر في الثانية، و5.6٪ تعمل بين 0.5 – 2 متر في الثانية و0.5٪ تجاوزت 2 متر في الثانية. عند انخفاض الطلب، سجل أكثر من نصف العقد شحنة ضغط ماء أقل من 7 متر، بينما تجاوز 25٪ هذه القيمة. لا توجد اختلافات كبيرة في مستويات السرعة أثناء انخفاض الطلب مقارنة باستخدامات ساعات الذروة. في المنطقة 835، سجّلت جميع العقد ضغطًا أقل من 7 أمتار ماء خلال ذروة الطلب، مع تسجيل حوالي 50% منها ضغطًا سالبًا. وأشار تحليل انخفاض الطلب إلى أن 80 عقدة تعمل تحت ضغط ماء اقل من 7متر، بينما تجاوزت 27 عقدة هذا المعيار. وكشف تحليل سرعة المياه أن 83% من الأنابيب تعمل تحت 0.5 متر/ثانية، بينما تتراوح نسبة 17% المتبقية بين 0.5 و2 متر/ثانية. يمكن الاستنتاج ان كلتا الشبكتين في الغالب عملت تحت مستوى الضغط المقبول وهو 7 متر ماء. وهذا يعني نقص في إمدادات المياه للمستهلكين، لاسيما خلال فترات ذروة الطلب.

الكلمات المفتاحية: معايرة دارون،QGIS ، التحليل الهيدروليكي، شبكات توزيع المياه، Water-CAD.